APPENDIX H

GEOTECHNICAL ANALYSES

1. SUMMARY OF GEOTECHNICAL DESIGN PARAMETERS

2. SLOPE STABILITY ANALYSIS - GLOBAL ANALYSIS, SECTION A-A' AND SLOPE STABILITY ANALYSIS - CHEMICAL WASTE UNIT, SECTION B-B'

3. FOUNDATION AND WASTE SETTLEMENT

4. BEARING CAPACITY OF LANDFILL FOUNDATION

5. GEOCOMPOSITE DRAINAGE NET CAPACITY

6. PREVIOUS GEOTECHNICAL ANALYSES

Shaw®

APPENDIX H.1

SUMMARY OF GEOTECHNICAL DESIGN PARAMETERS

Shaw®

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 9/17/07

Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

Problem Statement:

Summarize the various geotechnical parameters for the various landfill and in-situ materials present at the Clinton Landfill No.3.

Given:

- ❑ Hydrogeologic and Design Drawings contained in this application.
- ❑ Hydrogeologic Investigation contained in this application.
- ❑ Geotechnical Studies and Laboratory Test Results for insitu soil materials (contained in a later section of Appendix H).
- ❑ Coduto, Donald P., *Geotechnical Engineering, Principles and Practices* (please refer to attached pages).
- ❑ Das, Braja M., *Geotechnical Engineering*, *Third Edition*. (Please refer to attached pages).
- ❑ Direct Shear Test results for geosynthetic interfaces from Clinton Landfill No. 3 (please refer to attached pages).
- ❑ Geotechnical laboratory test results for the chemical waste (please refer to attached pages).
- ❑ USGS National Seismic Hazard Mapping Project. http://geohazards.cr.usgs.gov/eq (please refer to attached pages).
- ❑ Geotechnical Design Model for Clinton Landfill No. 3 (please refer to attached pages).

Assumptions:

The landfill will include various components as detailed below:

- ❑ Final Cover System
  - Vegetative cover (3 feet)
  - Geocomposite drainage net and filter geotextile
  - 40-mil HDPE textured geomembrane
  - 1-foot compacted final cover barrier layer ($k \leq 1\times10^{-7}$ cm/sec)
- ❑ Municipal Solid Waste

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: PCT Date: 9/17/07
Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

- ❑ Chemical Waste
- ❑ Leachate Collection System
  - 6 oz/yd^2 nonwoven cushion geotextile filter
  - 1-foot granular drainage layer ($k \geq 3.0 \times 10^{-2}$ cm/sec)
- ❑ Base Liner System
  - 60-mil HDPE textured geomembrane
  - Geosynthetic clay liner
  - 60-mil HDPE textured geomembrane
  - Geocomposite
  - 60-mil HDPE textured geomembrane
  - 3-foot low permeability compacted cohesive earth liner ($k \leq 1 \times 10^{-7}$ cm/sec).
  - Compacted clay fill replacing Roxana Silt / Robein Member beneath compacted cohesive earth liner (thickness varies)

Note, the proposed inverts of the Clinton Landfill Chemical Waste Unit will be located within the Roxana Silt and Robein Member; however, this soil will be removed down to the Berry Clay and Radnor Till Formation and replaced with recompacted clay soil comprised of site borrow materials that will include Tiskilwa and Berry Clay/Radnor Till soils.

- ❑ Sideslope Liner System
  - 18-inch layer of random fill or select fill
  - Geocomposite
  - 60-mil HDPE textured geomembrane
  - Geocomposite
  - 60-mil HDPE textured geomembrane
  - 3-foot low permeability compacted cohesive earth liner ($k \leq 1 \times 10^{-7}$ cm/sec).
- ❑ In-Situ/Foundation Soils
  - Tiskilwa Formation Glacial Till
  - Roxana Silt and Robein Member
  - Berry Clay and Radnor Till
  - Mahomet Sand Member (> 180 ft. below ground surface)
  - Pennsylvanian Bond Formation

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 9/17/07

Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

Material Properties:

In-Situ/Foundation Units

Strength Parameters

Soil strength parameters used were obtained from past geotechnical studies and laboratory testing performed on representative soil samples taken from the Clinton Landfill project site. Listed below is a summary of the soil material properties and strength parameters for the four uppermost geologic units occurring at the Clinton Landfill project site:

- ❑ Tiskilwa Formation: Brown and gray silty clay (CL), with localized sand strata;
- ❑ Roxana Silt and Robein Member: Dark brown organic silt (OL) and Peat (Pt);
- ❑ Berry Clay and Radnor Till: Gray clay (CL); and
- ❑ Mahomet Sand Member: Sand and gravel (SP / GP).

The lower geologic units (including the Pennsylvanian Bond Formation) are significantly deeper and therefore were not considered in the geotechnical analyses.

The short-term (undrained / total stress) and long-term (drained / effective stress) conditions are related to each other by the effective stress concept. The effective stress of a soil is the total stress minus the hydrostatic pore pressure. Since the soil is undrained during short-term conditions, the total stress strength parameters are used, and during long-term conditions effective stress strength parameters are used since the soil is assumed to be drained. Soil densities were calculated using soil/volume equations listed below. Listed below is a summary of the soil properties and strength parameters for the geologic units occurring at the Clinton Landfill project site.

$$g_{moist} = g_{dry}\,(1 + w) \qquad g_{sat} = \frac{(G_{soil} + e)\,g_{water}}{1 + e} \qquad e = \frac{(G_{soil})(g_{water})}{g_{dry}} - 1$$

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 9/17/07

Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

Geological Unit	Moist Unit Weight γ_{moist} (pcf)	Saturated Unit Weight γ_{sat} (pcf)	Short-Term Conditions (Undrained / Total Stress Conditions)		Long-Term Conditions (Drained / Effective Stress Conditions)	
			Cohesion c (psf)	Friction Angle ϕ (degrees)	Cohesion c' (psf)	Friction Angle ϕ' (degrees)
Tiskilwa Formation	135	140	3,500	0	600	26
Roxana Silt / Robein Member	100	110	1,400	0	1,500	19
Berry Clay / Radnor Till	140	148	6,000	0	1,100	18
Mahomet Sand Member	135	140	0	35	0	35

Consolidation Parameters

The initial void ratio (e_o), compression index (C_c), recompression index (C_{cr}),and pre-consolidation stress (P_c) for primary settlement were determined from laboratory test data as presented later in Appendix H. Values for the secondary compression index (C'_α) were estimated using Table 11.4 from Coduto . The consolidation parameters are summarized below.

Geological Unit	Void Ratio (e_o)	Compression Index (C_c)	Recompression Index (C_{cr})	Preconsolidation Stress (P_c)	Secondary Compression Index (C'_α)
Tiskilwa Formation	0.34	0.12	0.015	6,000 psf	0.0048
Roxana Silt / Robein Member	1.20	0.45	0.06	10,000 psf	0.0225
Berry Clay / Radnor Till	0.32	0.10	0.016	35,000 psf	0.004

Notes:
Secondary Compression was calculated using the referenced Coduto - Table 11.4. Specifically, the following correlations were used: C_α / C_c = 0.04 for inorganic clays and silts — the Tiskilwa Formation and Berry Clay / Radnor Till; and C_α / C_c = 0.05 for organic clays and silts — the Roxana Silt / Robein Member.

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

Based on the consolidation test results, it appears that the three geological units above are overconsolidated.

Final Cover System

Protective and Vegetative Soil Layer

The protective and vegetative soil layer will be obtained from on-site excavated soils. For simplicity in the slope stability analysis the final cover was modeled as one unit. The shear strength parameters for the final cover were determined from the results of the triaxial compression laboratory tests performed on representative site borrow soil samples (presented later in Appendix H). The unit weights and shear strength parameters for the final cover are summarized below.

Layer	Moist Unit Weight γ_{moist} (pcf)	Saturated Unit Weight γ_{sat} (pcf)	Short-Term Conditions (Undrained / Total Stress Conditions)		Long-Term Conditions (Drained / Effective Stress Conditions)	
			Cohesion c (psf)	Friction Angle ϕ (degrees)	Cohesion c' (psf)	Friction Angle ϕ' (degrees)
Final Cover	128	134	1,350	0	1,300	22

Final Cover System Geosynthetics

The final cover system contains a protective and vegetative soil layer underlain by a composite liner system that includes a double-sided geocomposite, a 40-mil textured HDPE geomembrane, and a 12-inch final cover barrier layer. For the infinite final cover slope stability analysis, the most critical failure surface is located between the textured geomembrane and the compacted soil layer.

A value of 24 degrees was used for the shear strength friction angle between the textured HDPE and the low permeability soil liner. Prior to construction of the final cover, the interface friction angles and cohesion values between each proposed construction material, will be verified under saturated conditions (worst case) using methods, such as "Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method" (ASTM D 5321).

Chemical Waste and Municipal Solid Waste

Unit Weight

The unit weight of the landfill (waste) will vary because of differences in waste constituents, degree of compaction efforts, height of waste placement, amount of daily cover placed, etc. The Chemical

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Shaw® Shaw Environmental, Inc. Calculated By: PCT Date: 9/17/07

Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

---

waste cells will be filled with chemical waste up to a maximum elevation of 812 feet MSL, and during a subsequent filling phase of the landfill, filling with municipal solid waste will occur over the two chemical waste cells up to a maximum elevation of approximately 844 feet MSL.

Chemical Waste. The unit weight of the chemical waste will vary because of moisture content, the particle size of the soil material, and the degree of compaction efforts. Standard Proctor laboratory test data indicate moist unit weights ranging from 77.1 pcf to 83.4 pcf (a copy of the test data is presented in the attached pages). A conservative value of 90 pcf was used in the geotechnical engineering analyses to account for waste consolidation and water saturation.

Municipal Solid Waste. The unit weight of municipal solid waste (MSW) will vary widely because of differences in waste stream constituents, the state of decomposition, and degree of compaction efforts. Published test data on MSW indicate unit weights ranging from 55 pcf to 75 pcf (a copy of the reference source is presented later in Appendix H). A conservative value of 75 pcf was used in the geotechnical engineering analyses to account for waste consolidation and water saturation.

Shear Strength

The geotechnical parameters for the chemical waste as presented in the table below, are based on field and laboratory test data (refer to attached pages). The geotechnical parameters for the MSW as presented in the table below, are deemed conservative and are values used for the permitted areas of the existing Clinton Landfill (refer to referenced sources presented later in Appendix H). It was assumed that both the Chemical waste and MSW have the same shear strength parameters for both the short-term and long-term conditions. This is conservative since over time the waste will increase in strength as the waste consolidates. The unit weight and shear strengths are summarized below.

Client:	Clinton Landfill, Inc.
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
Calculated By: PCT	Date: 9/17/07
Checked By: JPV	Date: 9/26/07

Shaw® Shaw Environmental, Inc.

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

Layer	Moist Unit Weight γ_{moist} (pcf)	Saturated Unit Weight γ_{sat} (pcf)	Short-Term Conditions[1] (Undrained / Total Stress Conditions)		Long-Term Conditions[2] (Drained / Effective Stress Conditions)	
			Cohesion c (psf)	Friction Angle ϕ (degrees)	Cohesion c' (psf)	Friction Angle ϕ' (degrees)
Chemical Waste	90	90	900	0	240	45
			840	17.4		
MSW	75	75	500	30	500	30

Notes:

1. Short Term Shear Strength values for the Chemical Waste:

- cohesion = 900 psf, and friction angle = 0 deg., reflect the total shear strength values from unconsolidated undrained triaxial shear strength testing performed on representative samples of the chemical waste to represent short term active waste fill conditions (at 3H:1V slope, maximum active face waste fill height of 50 feet); and
- cohesion = 840 psf, and friction angle 17.4 = 0 deg., reflect the total shear strength values from consolidated undrained triaxial shear testing performed on representative samples of the chemical waste to represent interim waste fill conditions (i.e., under maximum loading - maximum waste height / consolidated conditions).

2. Long Term Shear Strength values for the Chemical Waste reflect the effective shear strength values from consolidated undrained triaxial shear strength testing performed on representative samples of the chemical waste to represent long term waste fill conditions.

Leachate Collection System

The leachate collection system will consist of a 1 foot thick granular drainage blanket on the base of the landfill and an 18-inch layer of select fill on the sideslopes underlain by a geocomposite drainage net. The leachate collection system will be along the base will consist of materials with a hydraulic conductivity greater than or equal to 3.0×10^{-2} cm/sec. The unit weight and strength parameters are typical values used at landfills. The unit weight and strength parameters for the leachate collection system are summarized below.

Client: Clinton Landfill, Inc.

Project: Clinton Landfill No. 3 Chemical Waste Unit

Proj. #: 128017

Shaw® Shaw Environmental, Inc.

Calculated By: PCT Date: 9/17/07

Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

Layer	Moist Unit Weight γ_{moist} (pcf)	Saturated Unit Weight γ_{sat} (pcf)	Short-Term Conditions (Undrained / Total Stress Conditions)		Long-Term Conditions (Drained / Effective Stress Conditions)	
			Cohesion c (psf)	Friction Angle ϕ (degrees)	Cohesion c' (psf)	Friction Angle ϕ' (degrees)
Leachate Drainage Layer on Base	126	130	0	30	0	30
Compacted Random Fill on Sideslopes	128	134	1,350	0	1,300	22

Composite Base and Sideslope Liner System

<u>Compacted Cohesive Earth Liner and Compacted Clay Fill Sub-Base</u>

The compacted cohesive earth liner and compacted clay fill sub-base ("compacted clay fill") will be constructed using excavated in-situ soils from theTiskilwa Formation and Berry Clay / Radnor Till Unit. The shear strength parameters for the compacted cohesive earth liner and compacted clay fill were determined from the results of the triaxial compression laboratory tests performed on representative soil samples from project site borrow sources and prior CQA testing at the landfill site (refer to data presented later in Appendix H). The unit weights and shear strength parameters for the compacted cohesive earth liner and compacted clay fill are summarized below.

Layer	Moist Unit Weight γ_{moist} (pcf)	Saturated Unit Weight γ_{sat} (pcf)	Short-Term Conditions (Undrained / Total Stress Conditions)		Long-Term Conditions (Drained / Effective Stress Conditions)	
			Cohesion c (psf)	Friction Angle ϕ (degrees)	Cohesion c' (psf)	Friction Angle ϕ' (degrees)
Compacted Cohesive Earth Liner and Compacted Clay Fill / Sub-base	135	140	3,000	0	700	21

Shaw® Shaw Environmental, Inc.

Client:	Clinton Landfill, Inc.
Project:	Clinton Landfill No. 3 Chemical Waste Unit
Proj. #:	128017
Calculated By: PCT	Date: 9/17/07
Checked By: JPV	Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

The strength parameters for the short-term (undrained) and long-term (drained) conditions were assumed equal to the effective strength and total strength values, respectively, determined from the laboratory triaxial compression test results.

Consolidation Parameters

The consolidation parameters for the recompacted cohesive soil liner are taken from the previous laboratory testing performed on the project site soils (refer to data presented later in Appendix H). The consolidation parameters are summarized below.

Layer	Void Ratio (e_o)	Compression Index (C_c)	Recompression Index (C_{cr})	Secondary Compression Index (C'_α)
Compacted Cohesive Earth Liner and Compacted Clay Fill / Sub-base	0.32	0.10	0.016	0.004
Notes: Secondary Compression was calculated using the referenced Coduto - Table 11.4. Specifically, the following correlation was used: $C_\alpha / C_c = 0.04$ for inorganic clays and silts.				

Bottom Liner System Geosynthetics

The bottom composite liner system to be constructed on the base and sideslopes of the landfill will consist of a series of geosynthetics overlying a 3-foot thick compacted cohesive earth liner. The composite liner system from top to bottom for the base liner system is as follows:

- Base Liner System
  - 60-mil HDPE textured geomembrane
  - Geosynthetic clay liner
  - 60-mil HDPE textured geomembrane
  - Geocomposite
  - 60-mil HDPE textured geomembrane
  - 3-foot low permeability compacted cohesive earth liner ($k \leq 1\times10^{-7}$ cm/sec).
  - Compacted clay fill replacing Roxana Silt / Robein Member beneath compacted cohesive earth liner (thickness varies)

Note, the proposed inverts of the Clinton Landfill Chemical Waste Unit will be located within the Roxana Silt and Robein Member; however, this soil will be removed down to the Berry Clay and Radnor Till Formation and replaced with recompacted clay soil comprised of site borrow materials that will include Tiskilwa and Berry Clay/Radnor Till soils.

Shaw® Shaw Environmental, Inc.

Client: Clinton Landfill, Inc.
Project: Clinton Landfill No. 3 Chemical Waste Unit
Proj. #: 128017
Calculated By: PCT Date: 9/17/07
Checked By: JPV Date: 9/26/07

TITLE: SUMMARY OF GEOTECHNICAL PARAMETERS

The composite liner system from top to bottom for the sideslope liner system is as follows:

- ❑ Sideslope Liner System
  - Geocomposite
  - 60-mil HDPE textured geomembrane
  - Geocomposite
  - 60-mil HDPE textured geomembrane
  - 3-foot low permeability compacted cohesive earth liner ($k \leq 1 \times 10^{-7}$ cm/sec).

The stability of the composite liner will depend on interface shear strengths between liner components. The critical interface of the composite liner system is between the geomembrane and the recompacted soil liner or between the geotextile and the geomembrane. The proposed liner design utilizes a textured HDPE geomembrane on both the base and sideslopes of the liner system.

A value of 24 degrees was used for the interface friction angle between the textured HDPE geomembrane and the recompacted soil liner based on direct shear tests performed on site soil materials. On-site testing as required by the CQA report will be used to demonstrate that these strength parameters can be met. If on-site testing determines that these critical strength parameters can not be met, alternate landfill liner designs will be used.

Seismic Coefficients

Bedrock Acceleration

A seismic impact zone is defined as an area that has a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material will exceed 0.1 g in 250 years. From the United State Geologic Survey (USGS) Earthquake Hazards Program - National Seismic Hazard Mapping website, a site specific seismic coefficient for the Clinton Landfill was found to be 0.0981 g. A value of 0.0981 g was used in all seismic geotechnical analyses (refer to attached pages).

Free Field Acceleration

Seismic acceleration values obtained from the USGS are peak bedrock acceleration. Depending on the overlying materials, the bedrock acceleration may amplify, de-amplify, or stay the same. Since the soils overlying the bedrock are overconsolidated, therefore, classified as stiff soils, the RCRA Subtitle D (258) seismic design guidance for municipal solid waste landfill facilities recommends:

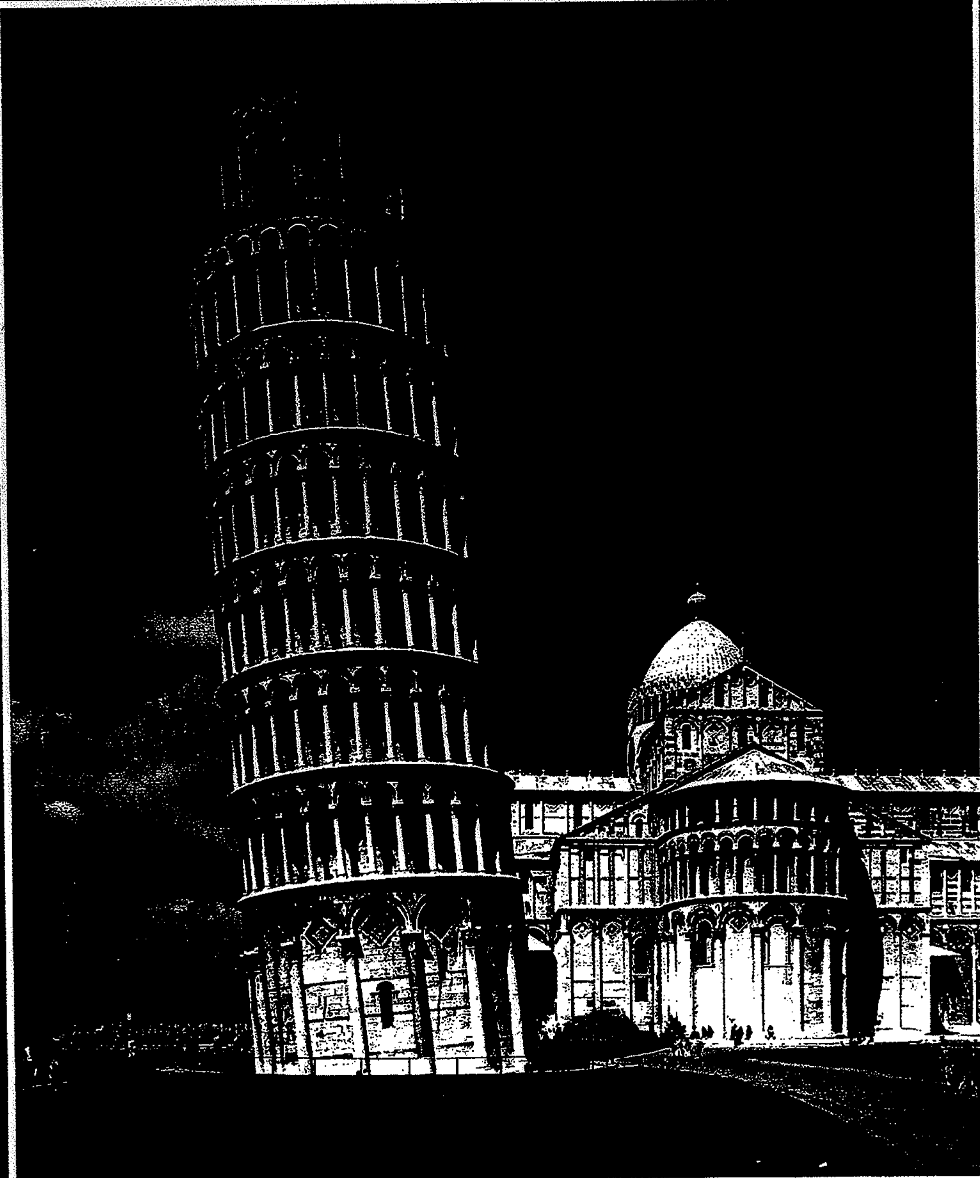
> ". . . for stiff sites for all acceleration levels, assume the free field peak ground acceleration at the site is equal to the peak bedrock acceleration."

Published References

Shaw®

GEOTECHNICAL ENGINEERING

Principles and Practices



DONALD P. CODUTO

where V_{v0} and V_{v1} are the initial and final void volumes, respectively. From the definition of void ratio,

$$\Delta V_v = \Delta e V_s \tag{8.16}$$

where Δe = change of void ratio. But

$$V_s = \frac{V_0}{1 + e_O} = \frac{AH}{1 + e_O} \tag{8.17}$$

where e_O = initial void ratio at volume V_0. Thus, from Eqs. (8.14), (8.15), (8.16), and (8.17),

$$\Delta V = SA = \Delta e V_s = \frac{AH}{1 + e_O} \Delta e$$

or

$$S = H \frac{\Delta e}{1 + e_O} \tag{8.18}$$

For normally consolidated clays that exhibit a linear e-log p (Figure 8.12) relationship,

$$\Delta e = C_c[\log (p_O + \Delta p) - \log p_O] \tag{8.19}$$

where C_c = slope of the e-log p plot and is defined as the compression index. Substitution of Eq. (8.19) in Eq. (8.18) gives

$$\boxed{S = \frac{C_c H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_O} \right)} \tag{8.20}$$

For a thicker clay layer, it is more accurate if the layer is divided into a number of sublayers and calculations for settlement are made separately for each sublayer. Thus, the total settlement for the entire layer can be given as

$$S = \sum \left[\frac{C_c H_i}{1 + e_O} \log \left(\frac{p_{O(i)} + \Delta p_{(i)}}{p_{O(i)}} \right) \right]$$

where H_i = thickness of sublayer i
$p_{O(i)}$ = initial average effective overburden pressure for sublayer i
$\Delta p_{(i)}$ = increase of vertical pressure for sublayer i

In overconsolidated clays (Figure 8.13), for $p_O + \Delta p \le p_c$ field e-log p variation will be along the line cb, the slope of which will be approximately equal to that for the laboratory rebound curve. The slope of the rebound curve, C_s, is referred to as the *swell index*, so

$$\Delta e = C_s[\log (p_O + \Delta p) - \log p_O] \tag{8.21}$$

From Eqs. (8.18) and (8.21),

$$\boxed{S = \frac{C_s H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_O} \right)} \tag{8.22}$$

If $\boxed{p_O + \Delta p} > \boxed{p_c}$ then $\sigma_o' + \Delta\sigma' > \sigma_c'$

$$\boxed{S = \frac{C_s H}{1 + e_O} \log \frac{p_c}{p_O} + \frac{C_c H}{1 + e_O} \log \left(\frac{p_O + \Delta p}{p_c} \right)} \tag{8.23}$$

However, if the e-log p curve is given, it is possible simply to pick Δe off the plot for the appropriate range of pressures. This figure may be substituted into Eq. (8.18) for the calculation of settlement, S.

8.8 COMPRESSION INDEX (C_c)

The compression index for the calculation of field settlement caused by consolidation can be determined by graphic construction (as shown in Figure 8.12) after obtaining laboratory test results for void ratio and pressure.

Terzaghi and Peck (1967) suggested the following empirical expressions for compression index:

For undisturbed clays:

$$C_c = 0.009(LL - 10) \tag{8.24}$$

For remolded clays:

$$C_c = 0.007(LL - 10) \tag{8.25}$$

where LL = liquid limit, in percent.

In the absence of laboratory consolidation data, Eq. (8.24) is often used for an approximate calculation of primary consolidation in the field.

Several other correlations for the compression index are also available now. They have been developed by tests on various clays. Some of these correlations are given in Section E.2 (Appendix E).

The secondary compression index can be defined from Figure 8.22 as

$$\boxed{C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log (t_2/t_1)}} \tag{8.30}$$

where C_α = secondary compression index
Δe = change of void ratio
t_1, t_2 = time

The magnitude of the secondary consolidation can be calculated as

$$\boxed{S_s = C'_\alpha H \log \left(\frac{t_2}{t_1}\right)} \tag{8.31}$$

where

$$\boxed{C'_\alpha = \frac{C_\alpha}{1 + e_p}} \tag{8.32}$$

e_p = void ratio at the end of primary consolidation (Figure 8.22)
H = thickness of clay layer

The general magnitudes of C'_α as observed in various natural deposits are given in Figure 8.23.

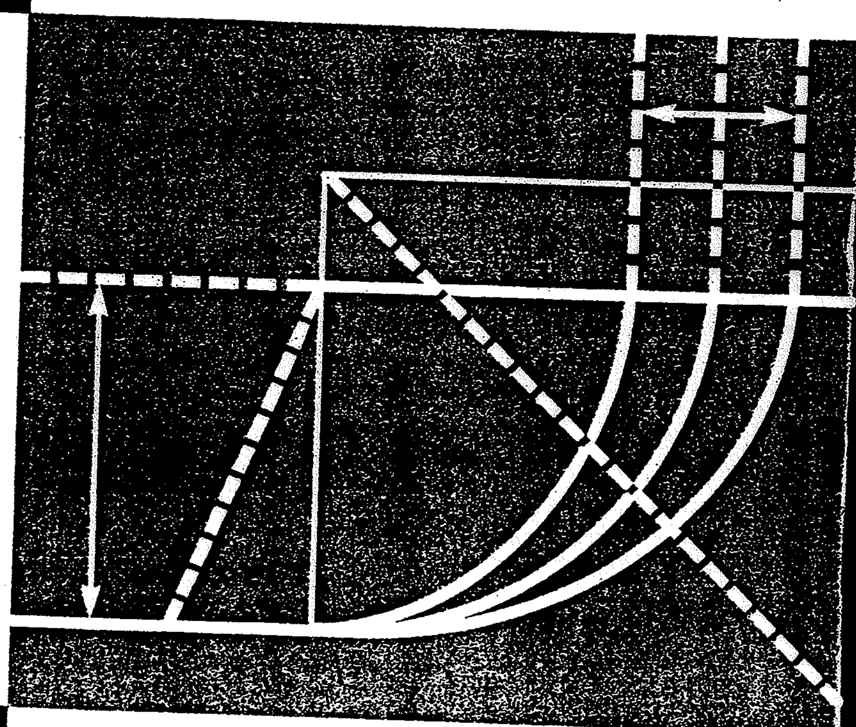
Secondary consolidation settlement is more important than primary consolidation in organic and highly compressible inorganic soils. In overconsolidated inorganic clays, the secondary compression index is very small and of less practical significance.

There are several factors that might affect the magnitude of secondary consolidation, some of which are not very clearly understood (Mesri, 1973). The ratio of secondary to primary compression for a given thickness of soil layer is dependent on the ratio of the stress increment (Δp) to the initial effective stress (p). For small $\Delta p/p$ ratios, the secondary-to-primary compression ratio is larger.

B R A J A   M. D A S

Principles of Geotechnical Engineering

Third Edition



secondary compression and occurs under a constant effective stress. We don't fully understand the physical basis for secondary compression, but it appears to be due to particle rearrangement, creep, and the decomposition of organics. Highly plastic clays, organic soils, and sanitary landfills are most likely to have significant secondary compression. However, secondary compression is negligible in sands and gravels.

The *secondary compression index*, C_α, defines the rate of secondary compression. It can be defined either in terms of either void ratio or strain:

$$\boxed{C_\alpha = -\frac{de}{d\log t}} \tag{11.26}$$

$$\boxed{\frac{C_\alpha}{1+e_p} = \frac{d\epsilon_z}{d\log t}} \tag{11.27}$$

where:

C_α = secondary compression index

e = void ratio

e_p = void ratio at end of consolidation settlement (can use $e_p = e_0$ without introducing much error)

ϵ_z = vertical strain

t = time

Design values are normally determined while conducting a laboratory consolidation test. The consolidation settlement occurs very rapidly in the lab (because of the short drainage distance), so it is not difficult to maintain one or more of the load increments beyond the completion of consolidation settlement. The change in void ratio after this point can be plotted against log time to determine C_α.

Another way of developing design values of C_α is to rely on empirical data that relates it to the compression index, C_c. This data is summarized in Table 11.4.

TABLE 11.4 EMPIRICAL CORRELATION BETWEEN C_α AND C_c (Terzaghi, Peck, and Mesri, 1996)

Material	C_α/C_c
Granular soils, including rockfill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat and muskeg	0.06 ± 0.01

Direct Shear Test Results -
Interface Shear Strength Summary
Clinton Landfill No. 3

Shaw®

INTERFACE FRICTION TEST SUMMARY
Clinton Landfill No. 3
Testing conducted by Precision Analytical Laboratories, March, April, May, and September 2001

Description	Normal Stress (psf)	Shear Stress (psf) Peak	Shear Stress (psf) Residual	Design - Peak Strength Phi	Design - Peak Strength C (psf)	Design - Resdual Strength Phi	Design - Resdual Strength C (psf)
Final Cover Components							
Final cover soil vs. Geotextile	250	198	168	30	0	24	0
	350	210	175				
	450	328	310				
Geotexile vs. Geonet	250	181	149	28	30	25	0
	350	216	171				
	450	286	226				
Geonet vs. Textured HDPE	250	124	119	26	0	19	25
	350	215	208				
	450	240	194				
Final cover soil vs. Smooth HDPE	250	218	206	31	65	28	65
	350	278	262				
	450	339	316				
Liner Floor Components							
LCS Sand vs. Smooth HDPE	5000	2427	1940	20	450	10	900
	7500	3164	2134				
	10000	4300	2860				
Sidewall Liner - Prior to Waste Placement							
Protective cover soil vs. Geotextile	250	198	168	30	0	24	0
	350	210	175				
	450	328	310				
Geotextile vs. Geonet	250	181	149	28	30	25	0
	350	216	171				
	450	286	226				
Geomembrane vs. Geonet	100	96	81	10	75	8	65
	275	136	125				
	450	160	135				
Sidewall Liner - Following Waste Placement							
Geotextile vs. Geonet	2500	719	610	15	0	14	0
	5000	1351	1183				
	8000	2600	1975				
Geonet vs. Smooth HDPE	2500	622	622	11	126	11	126
	5000	1102	1070				
	8000	1702	1683				
Clay liner soil vs. Smooth HDPE	2500	1072	748	16	345	12	185
	5000	1765	1288				
	8000	2651	1950				
Clay liner soil vs. Geotextile	2500	1598	1528	24	400	24	380
	5000	2612	2582				
	8000	4112	3990				

Geotechnical Design Model - Clinton Landfill No. 3

Shaw®

GEOTECHNICAL DESIGN MODEL

Clinton Landfill No. 3

November 2006

SOIL STRATA	TOP / BOTTOM ELEV. (feet – MSL)	GEOTECHNICAL CHARACTERISTICS
BROWN AND GRAY SILTY CLAY (CL), with localized sand strata [Tiskilwa Formation]	Ground Surface / 670	Medium stiff to hard $\phi = 0$, $c = 3{,}500$ psf $\phi' = 26^0$, $c' = 600$ psf $\gamma_t = 135$ pcf, $\gamma_{sat} = 140$ pcf $P_c = 6{,}000$ psf $C_c = 0.12$, $C_{cr} = 0.015$, $e_0 = 0.34$
DARK BROWN ORGANIC SILT (OL) AND PEAT (Pt) [Roxana Silt / Robein Member]	670 / 660	Medium stiff to stiff $\phi = 0$, $c = 1{,}400$ psf $\phi' = 19^0$, $c' = 1{,}500$ psf $\gamma_t = 100$ pcf, $\gamma_{sat} = 110$ pcf $P_c = 10{,}000$ psf $C_c = 0.45$, $C_{cr} = 0.06$, $e_0 = 1.20$
GRAY CLAY (CL) [Berry Clay and Radnor Till]	660 / 483	Hard $\phi = 0$, $c = 6{,}000$ psf $\phi' = 18^0$, $c' = 1{,}100$ psf $\gamma_t = 140$ pcf, $\gamma_{sat} = 148$ pcf $P_c = 35{,}000$ psf $C_c = 0.10$, $C_{cr} = 0.016$, $e_0 = 0.32$
SAND AND GRAVEL (SP/GP) [Mahomet Sand Member]	< 483	Very dense (N > 50) $\phi' = 35^0$, $c' = 0$ psf $\gamma_t = 135$ pcf, $\gamma_{sat} = 140$ pcf

Note: This geotechnical design model is intended to conservatively represent prevalent subsurface conditions upon which the geotechnical design is based. Actual conditions will be observed by the CQA Officer during construction to verify that the design parameters are appropriate. The following pages document the development of this model.

T'SKILWA FORMATION

Strength Parameters

Unconsolidated – Undrained Shear Strength (short-term behavior)

Unconfined Compression Test Results:

Boring	Depth / Elevation (ft)	Unconfined Compressive Strength (tsf)
EX-10	12.5	4.0
EX-10	31.0	4.2
EX-10	39.0	2.2
EX-11	Elev. 691.2	3.7
	Average	3.53

Triaxial Shear (unconsolidated – undrained) Test Results:

Boring	Depth / Elevation (ft)	Deviator Stress (Shear Strength) (tsf)
CLG 1 & 2	12	1.38
CLG 1 & 2	12	2.34
CLG 1 & 2	12	3.17
CLG 3 & 4	38	1.93
CLG 3 & 4	38	3.12
CLG 3 & 4	38	3.43
EX-13	12	10.99
EX-13	12	4.90
EX-13	42	14.69
EX-17	19.5	2.51
EX-17	29.5	20.69
EX-19	45	8.87
EX-21	13	6.81
EX-21	31	1.03
EX-23	34.5	11.91
EX-24	11	14.46
EX-24	31	11.51
EX-24	55	10.75
	Average:	7.47

Field Pocket Penetrometer Data:

Ranges from ~0.5 tsf to >4.5 tsf, majority between 2.0 and 4.0 tsf

Φ = 0 (assumed)

C = Shear Strength / 2

Shear strength = 3.5 tsf based on lab and field tests

C = 3.5 tsf / 2

C = 1.75 tsf = 3,500 psf

<u>Consolidated – Drained Shear Strength (long-term behavior)</u>

See attached Figure 1.

Φ' = 26^0

C' = 0.3 tsf = 600 psf

<u>Density</u>

Boring	Depth / Elevation (ft)	Dry Density (pcf)	Moisture Content (%)	Total Density (pcf)
EX-10	12.5	124.2	10.8	138
EX-10	31.0	117.0	13.2	132
EX-10	39.0	111.5	14.0	127
EX-11	691.2	125.0	12.0	140
EX-13	12.0	121.9	12.1	137
EX-17	4.0	117.4	16.0	136
EX-17	14.0	122.6	13.9	140
EX-17	19.0	117.4	16.0	136
EX-21	31.0	122.8	13.3	139
EX-24	11.0	117.9	14.8	135
EX-24	55.0	124.2	13.0	140
			Average:	136.36

To be conservative, use **total unit weight = 135 pcf**

Saturated unit weight = 140 pcf

Consolidation Parameters

Consolidation Test Results Summary:

Boring	Depth / Elevation (ft)	e_0	C_c	C_{cr}	P_c (tsf)
EX-10	12.5	0.331	0.113	0.013	2.9
EX-11	Elev. 691.2	0.348	0.125	0.017	5.2
EX-19	Elev. 674	0.281	0.080	0.016	3.0
EX-23	Elev. 676	0.320	0.105	0.018	4.2
		0.32	0.106	0.016	3.83

For conservative design purposes, use:

$e_0 = 0.34$

$C_c = 0.12$

$C_{cr} = 0.015$

$P_c = 3.0$ tsf

XANA / ROBEIN SILT

Strength Parameters

Unconsolidated – Undrained Shear Strength (short-term behavior)

Unconfined Compression Test Results:

Boring	Depth / Elevation (ft)	Unconfined Compressive Strength (tsf)
EX-6A	24.5	1.8
EX-6A	26.5	1.6
EX-6A	28.5	1.0
EX-6B	24.5	1.4
EX-6B	26.5	1.3
EX-6B	28.5	1.05
	Average	1.36

Triaxial Shear (unconsolidated – undrained) Test Results:

Boring	Depth / Elevation (ft)	Deviator Stress (Shear Strength) (tsf)
CLG 5	40	0.29
CLG 5	40	0.40
CLG 5	40	0.45
EX-13	52	8.17
EX-16	49.5	1.47
EX-19	55	6.96
	Average:	2.96

Φ = 0 (assumed)

C = Shear Strength / 2

Shear strength = 1.4 tsf based on lab tests

C = 1.4 tsf / 2

C = 0.7 tsf = 1,400 psf

Consolidated – Drained Shear Strength (long-term behavior)

See attached Figure 2.

Φ' = 19^0

C' = 0.75 tsf = 1,500 psf

Density

Boring	Depth / Elevation (ft)	Dry Density (pcf)	Moisture Content (%)	Total Density (pcf)
EX-6A	26.5	67.0	46.9	98
EX-6B	26.5	70.1	42.7	100
EX-16	49.5	70.2	46.4	103
EX-19	54	66.7	47.5	98
			Average:	99.75

To be conservative, use **total unit weight = 100 pcf**

Consolidation Parameters

Consolidation Test Results Summary:

Boring	Depth / Elevation (ft)	e_0	C_c	C_{cr}	P_c (tsf)
EX-6A	26.5	1.013	0.186	0.085	5.0
EX-6B	26.5	0.922	0.178	0.025	5.5
EX-16	Elev. 667.5	1.136	0.443	0.050	7.9
EX-23	Elev. 664	1.487	0.490	0.070	5.8
		1.14	0.324	0.058	6.05

For conservative design purposes, use:

e_0 = 1.20

C_c = 0.45

C_{cr} = 0.06

P_c = 5.0 tsf

BERRY CLAY / RADNOR TILL

Strength Parameters

Unconsolidated – Undrained Shear Strength (short-term behavior)

Unconfined Compression Test Results:

Boring	Depth / Elevation (ft)	Unconfined Compressive Strength (tsf)
EX-4	64	9.17
EX-4	79	8.63
EX-5	69	9.07
EX-5	79	9.52
EX-5	90.5	10.14
EX-6	32	7.16
EX-6	49	13.90
EX-7	32	6.27
EX-7	47	13.90
	Average	9.75

Triaxial Shear (unconsolidated – undrained) Test Results:

Boring	Depth / Elevation (ft)	Deviator Stress (Shear Strength) (tsf)
EX-13	64	3.56
EX-16	72	2.41
EX-19	59	2.52
EX-19	69	19.97
EX-21	39	2.73
EX-23	60	6.15
EX-24	76	4.71
	Average:	6.01

Φ = 0 (assumed)

C = Shear Strength / 2

Shear strength = 6.0 tsf based on lab tests

C = 6.0 tsf / 2

C = 3.0 tsf = 6,000 psf

Consolidated – Drained Shear Strength (long-term behavior)

See attached Figure 3.

Φ' = 18^0

C' = 0.55 tsf = 1,100 psf

Density

Boring	Depth / Elevation (ft)	Dry Density (pcf)	Moisture Content (%)	Total Density (pcf)
EX-4	64	135.1	12.6	152
EX-4	79	139.0	8.4	151
EX-5	69	135.8	12.1	152
EX-5	79	144.7	7.8	156
EX-5	90.5	142.9	7.9	154
EX-6	32	114.6	18.4	136

EX-6	49	110.6	19.7	133
EX-7	32	118.4	17.6	139
EX-7	47	146.7	6.2	156
EX-13	62	109.1	18.7	130
EX-16	72	147.6	6.6	157
EX-19	59	113.6	16.2	132
EX-19	69	127.5	11.8	143
EX-21	39	112.9	16.8	132
EX-24	76	112.6	17.9	133
		Average:	13.25	143.73

Use total unit weight = 140 pcf

Consolidation Parameters

Consolidation Test Results Summary:

Boring	Depth / Elevation (ft)	e_0	C_c	C_{cr}	P_c (tsf)
EX-5	79	0.09	0.030	0.009	4.0
EX-13	Elev. 660	0.338	0.080	0.019	4.0
EX-21	Elev. 662	0.494	0.130	0.019	6.2
		0.307	0.08	0.016	4.73

The interpreted preconsolidation pressure from the consolidation tests appear incorrect. The likely reason is that the tests were not performed to high enough loads which surpass the preconsolidation pressure.

Check preconsolidation pressure based on Liquidity Index and geologic history.

Atterberg Limits Test Results Summary:

Boring	Depth / Elevation (ft)	Plastic Limit	Liquid Limit	Plasticity Index
B-23	66	19	31	12
B-21	54	18	26	8
B-11	88	18	27	9
EX-13	Elev. 660	11	37	25

EX-19	Elev. 658	13	30	17
EX-21	Elev. 662	13	38	25
EX-24	Elev. 660	12	39	27
EX-16	Elev. 645	11	16	5
EX-19	Elev. 649	11	20	9
EX-22	Elev. 664	11	32	21
EX-22	Elev. 659	9	27	18
EX-22	Elev. 654	10	30	21
EX-23	Elev. 648	6	18	12
	Average:	12.5	28.5	16.1

Liquidity Index = (Moisture Content – Plastic Limit) / Plasticity Index

Using average Moisture Content, PL and PI:

Liquidity Index = 0.04

Based on Figure 3, NAVFAC DM 7.1 – Preconsolidation Pressure vs. Liquidity Index (attached), the preconsolidation pressure could range from approx. 6.5 tsf to 25 tsf (13,000 psf to 50,000 psf).

The Berry Clay and Radnor Till are Illinoian glacial deposits. Therefore, they were overlain by Wisconsinan glaciers. According to notes from the Illinois State Geological Survey September 1997 and May 1998 field trips (attached), the Wisconsinan glaciers were approximately 700 feet thick over most of Illinois. Based on this, and a unit weight of ice (0.91 x 62.4 pounds per cubic foot), the preconsolidation pressure estimated from geologic history = 39,750 psf.

For conservative design purposes, use:

e_0 = 0.32

C_c = 0.10

C_{cr} = 0.016

P_c = 35,000 psf

C PACTED EARTH LINER / COMPACTED CLAY FILL

Strength Parameters

Unconsolidated – Undrained Shear Strength (short-term behavior)

Triaxial Shear (unconsolidated – undrained) Test Results – Remolded Composite Samples (remolded to 92 - 97% standard Proctor):

Source	Deviator Stress (Shear Strength) (tsf)
Weathered Tiskilwa	2.68
Weathered Tiskilwa	2.36
Weathered Tiskilwa	2.31
Unweathered Tiskilwa	6.34
Unweathered Tiskilwa	6.21
Unweathered Tiskilwa	5.43
Berry Clay	4.36
Berry Clay	4.50
Berry Clay	4.57
Average	4.31

Φ = 0 (assumed)

C = Shear Strength / 2

Shear strength = 3.0 tsf based on lab tests (conservative)

C = 3.0 tsf / 2

C = 1.5 tsf = 3,000 psf

Consolidated – Drained Shear Strength (long-term behavior)

See attached Figure 4.

Φ' = 21^0

C' = 0.35 tsf = 700 psf

Density

From Clinton Landfill Earth Liner CQA testing, typical total density of compacted Earth Liner = 135 pcf.

Use total unit weight = 135 pcf

Consolidation Parameters

From NAVFAC DM 7.2, Chapter 2, Table 1:

@ 1.4 tsf, compression = 1.3%

@3.6 tsf, compression = 2.5%

Use the attached graph to estimate % compression for other loads.

VEGETATIVE COVER AND INTERMEDIATE COVER

Strength Parameters

Unconsolidated – Undrained Shear Strength (short-term behavior)

Triaxial Shear (unconsolidated – undrained) Test Results – Remolded Composite Samples (remolded to 86 - 88% standard Proctor):

Source	Deviator Stress (Shear Strength) (tsf)
Weathered Tiskilwa	0.90
Weathered Tiskilwa	0.89
Weathered Tiskilwa	1.04
Unweathered Tiskilwa	2.36
Unweathered Tiskilwa	1.72
Unweathered Tiskilwa	2.90
Average	1.64

Φ = 0 (assumed)

C = Shear Strength / 2

Shear strength = 1.35 tsf based on lab tests

C = 1.35 tsf / 2

C = 0.675 tsf = 1,350 psf

Consolidated – Drained Shear Strength (long-term behavior)

See attached Figure 5.

$\Phi' = 22^0$

C' = 0.65 tsf = 1,300 psf

Density: **Use total unit weight = 128 pcf**

..NDFILLED WASTE

Strength Parameters

Non- to Partially Degraded Waste

From published and unpublished literature:

Source	phi	Cohesion (psf)
Sharma & Lewis (1994)	20^0	400
Duncan & Wright (2005)		
Kavazanjian et al. (1995[1])	33^0	0
Eid et al. (2000)	35^0	522
Stark (2005)	35^0	500
Stark & Huvaj-Sarihan (publication pending, 2006)		
$P_{ob} < 4200$ psf	35^0	125
$P_{ob} \geq 4200$ psf	30^0	625

Note 1. For overburden pressures (P_{ob}) greater than 775 psf)

For design use: **$\Phi = 30^0$**

C = 500 psf

Well Degraded Waste

From published and unpublished literature:

Source	phi	Cohesion (psf)
Duncan & Wright[1] (2005)	33^0	0
Stark (2005) and Stark & Huvaj-Sarihan (publication pending, 2006)[2]	20^0	0
Gabr, Hossain & Barlaz (publication pending, 2006)	24^0	0

Notes:

1. After Kavazanjian (1995 and 2001) for overburden pressures greater than 775 psf)
2. According to Stark & Huvaj-Sarihan "These parameters can be used to evaluate final slopes with leachate recirculation but should only be applied in areas that have undergone complete degradation, such as at or near the bottom of the waste mass."

For design use: **$\Phi = 22^0$**

C = 0 psf

Assume the lower 10 feet of waste is fully degraded during the landfill active phases (except during the initial two phases of landfill development). Assume the lower 24 feet of waste is fully degraded during the Post-Closure Period and thereafter.

<u>Density</u>:

<u>Non- to Partially Degraded Waste</u>

From article in *MSW Management* by R.T. Sprague and G.N. Richardson (Sept./Oct. 2001), MSW density versus depth can be estimated from density profile data from Puente Hills, California Landfill (after Kavazanjian, Matasovic, Bonaparte, and Schmertmann). Puente Hills Landfill waste is estimated to be at a moisture content of 22%. A plot of their data is attached.

Leachate recirculation will increase the moisture content of the landfilled waste. HELP modeling indicates an average waste moisture content throughout the Design Period of about 29%. Therefore, one must adjust the Puente Hills Landfill waste densities to account for the higher moisture contents expected at Clinton Landfill No. 3 using the following equation:

γ_{33} = unit weight at moisture content (w) = 29%

$= ((1 + 0.29) \times \gamma_{22}) / (1 + 0.22)$

$= 1.06 \times \gamma_{22}$

Depth Range (feet)	Average Unit Weight		
	W = 22%	W = 29%	
	(kN / m^3)	(kN / m^3)	(lbs / ft^3)
0 – 65	8.4	8.9	57
65 – 155	11.4	12.1	77
155 - 210	13.1	13.9	88

Weighted average γ_{33} = ((57 x 65) + (77 x 90) + (88 x 55)) / 210 = 74 pcf

According to Stark (2005), waste unit weight = 75 to 85 pcf

For design use **total density = 75 pcf** (inclusive of daily and intermediate cover)

Fully Degraded Waste

According to Stark (2005), fully degraded waste density = 100 to 110 pcf

For design use **total density = 105 pcf** (inclusive of daily and intermediate cover)

Waste Hydraulic Properties

References:

Geotechnical Aspects of Landfill Design and Construction; Qian, Koerner, and Gray; 2002

The Hydrologic Evaluation of Landfill Performance (HELP) Model, Engineering Documentation for Version 3; Schroeder, Dozier, Aappi, McEnroe, Sjostrom, and Peyton; EPA/600/R-94/168b; September 1994

The Hydrologic Evaluation of Landfill Performance (HELP) Model, User's Guide for Version 3; Schroeder, Lloyd and Zappi; EPA/600/R-94/168a; September 1994

Shear Strength of Municipal Solid Waste for Stability Analysis; Stark and Huvaj-Sarihan; unpublished, to be submitted for review and possible publication in the *Canadian Geotechnical Journal*; September 27, 2006

Initial Water Content

According to Qian, et.al. (Table 6.5, p. 188), the volumetric water content of landfilled waste varies from about 5% to 30%, with an average of 18.3%. Qian, et.al. also calculates the average dry gravimetric water content of waste as 26%. This is equivalent to 20% on a volumetric basis for a waste density of 60 pounds per cubic feet (pcf) and 25% on a volumetric basis for a waste density of 75 pcf. To be conservative, assume an initial volumetric water content of waste = 26% for the HELP Modelling.

Field Capacity

Waste at the bottom of the landfill is expected to become fully degraded with time (Stark and Huvaj-Sarihan). The overall unit weight of the landfilled waste mass (inclusive of daily and intermediate cover) is estimated to be 75 pounds per cubic foot (pcf). As the waste degrades, its density is estimated to increase to about 105 pcf (Stark and Huvaj-Sarihan). In addition to waste density, pertinent hydraulic properties of the waste used in the HELP Modeling (primarily field capacity and hydraulic conductivity) are also expected to change as the waste degrades.

The HELP Model default volumetric field capacity is 29.2% for a waste density of 900 pounds per cubic yard (33.3 pcf). According to Qian (p. 583), the field capacity increases as waste density increases. Waste densities using current placement and compaction equipment significantly exceeds 33.3 pounds per cubic foot. However, to be conservative, modeling of the landfill will be based on the HELP Model default volumetric field capacity of 29.2% for undegraded to partially degraded waste.

According to Stark and Huvaj-Sarihan, fully degraded waste has the consistency of clay (i.e. very small particles). Therefore, besides an increase in the waste density, the waste particle size is expected to decrease as the waste degrades. According to Qian (p. 583), an increase in density, and a decrease in particle size both increase the field capacity of waste. The HELP Model default volumetric field capacity for low density clays vary from 31% to 37.8%, with an average 35.0%. A field capacity of 31% is conservative for use in the HELP model for degraded waste.

Hydraulic Conductivity

The HELP Model default waste hydraulic conductivity = 1×10^{-3} cm/sec

(Although the HELP Model User's Guide indicates a waste density of 900 lb/yd^3, the HELP Model Engineering Documentation indicates that the default properties are for "well compacted municipal solid waste")

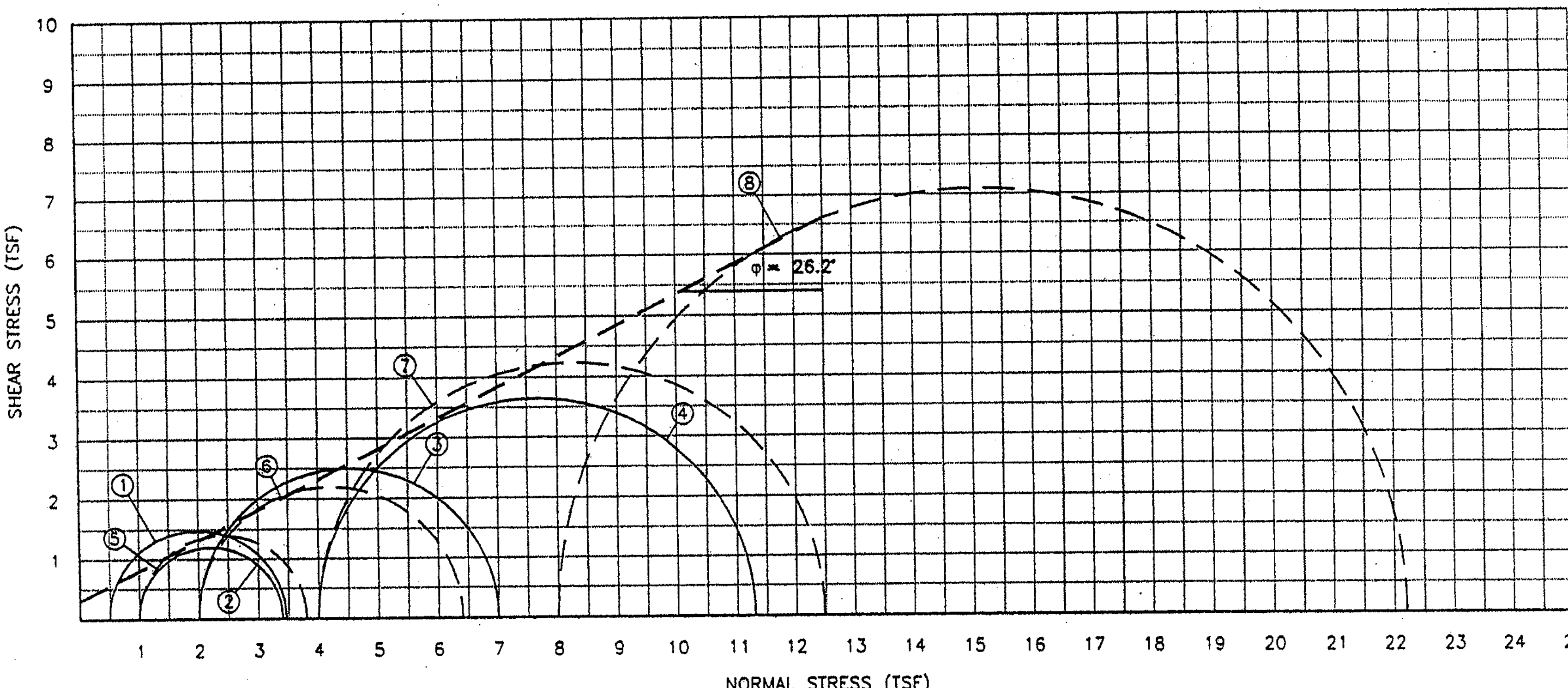
Qian, et.al. (Section 6.5) concludes that the "average hydraulic conductivity of municipal solid waste in landfills is approximately 1×10^{-3} cm/sec". This conclusion is based on data for waste densities varying from about 7 to 90 pcf. It is also noted that the data is from published sources dated 1994 and earlier. Heavier, more efficient waste compaction equipment, and leachate recirculation activities that have been used in more recent years would be expected to increase waste densities. Based on more recent data, Qian, et.al. (p. 185) concludes that waste densities are typically 55 to 70 pcf.

Figure 6.3 (Qian, et.al.) indicates a decreasing trend in waste hydraulic conductivity with increasing waste density. Plotting the median of their trend lines to a density typical of fully degraded waste (105 pcf or 16.4 kN/m^3) indicates a hydraulic conductivity equal to 0.8×10^{-5} m/sec (8.0×10^{-4} cm/sec). Therefore, a hydraulic conductivity for degraded waste equal to 8.0×10^{-4} cm/sec is appropriate for the HELP Modelling.

Porosity and Wilting Point

Waste porosity would be expected to increase, and wilting point would be expected to decrease as the waste degrades. Again, consistent with Stark and Huvaj-Sarihan's characterization of degraded waste as appearing similar to soft clay, use the average HELP Model default values for low density clay porosity and wilting point values (47.2% and 22.8%, respectively).

CAD DWG NO:911181108 ... O: 9834 DATE/TIME: 06/07/06 09:25 ACAD-15.



WEATHERED SAMPLE SOURCE	UNWEATHERED SAMPLE SOURCE
① EX-13 @12'	⑤ EX-13 @42'
② EX-17 @4'	⑥ EX-17 @14'
③ EX-21 @11'	⑦ EX-19 @44'
④ EX-24 @11'	⑧ EX-24 @31'

TISKILWA TILL
CONSOLIDATED DRAINED
RESIDUAL STRENGTH
(Long-Term Behavior)

$c = 0.3$

$\tan \varphi = \frac{3.25-0.3}{6.0-0}$

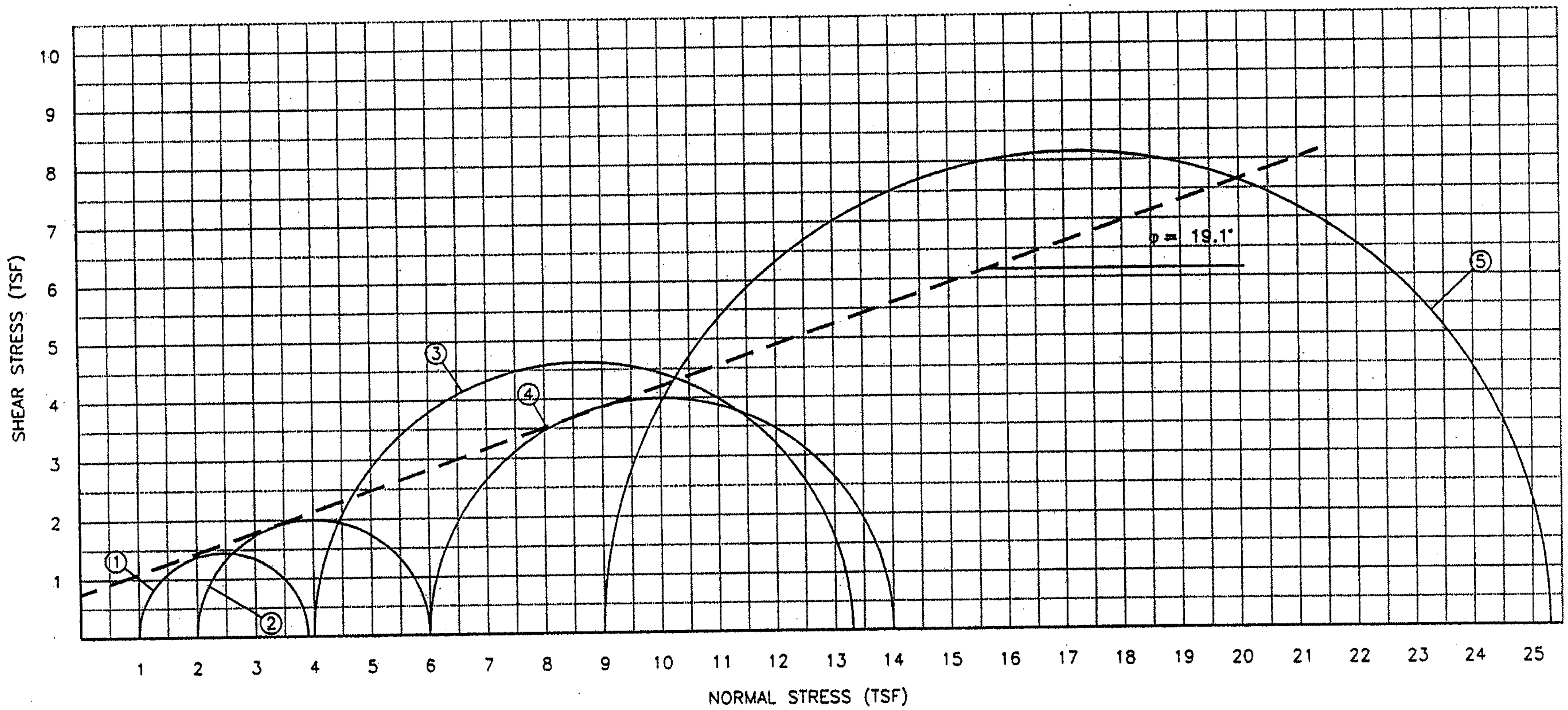
$\varphi = 26.2°$

Illinois Licensed Professional Design Firm 184-001145

PDC Technical Services, Inc.
Peoria, Illinois

FIGURE 1
SHEAR TEST DATA
CLINTON LANDFILL NO. 3
CLINTON, ILLINOIS
PROJECT NO. 91-118

CAD DWG NO:9111811086 NO: 9834 DATE/TIME: 10/25/06 10:55 ACAD-16.



SAMPLE SOURCE

① EX-24 @55'
② EX-21 @31'
③ EX-19 @54'
④ EX-16 @49'
⑤ EX-13 @52'

ROBEIN/ROXANA SILT
CONSOLIDATED DRAINED
RESIDUAL STRENGTH
(Long-Term Behavior)

$c = 0.75$

$\tan \varphi = \frac{4.9-.75}{12-0}$

$\varphi = 19.1°$

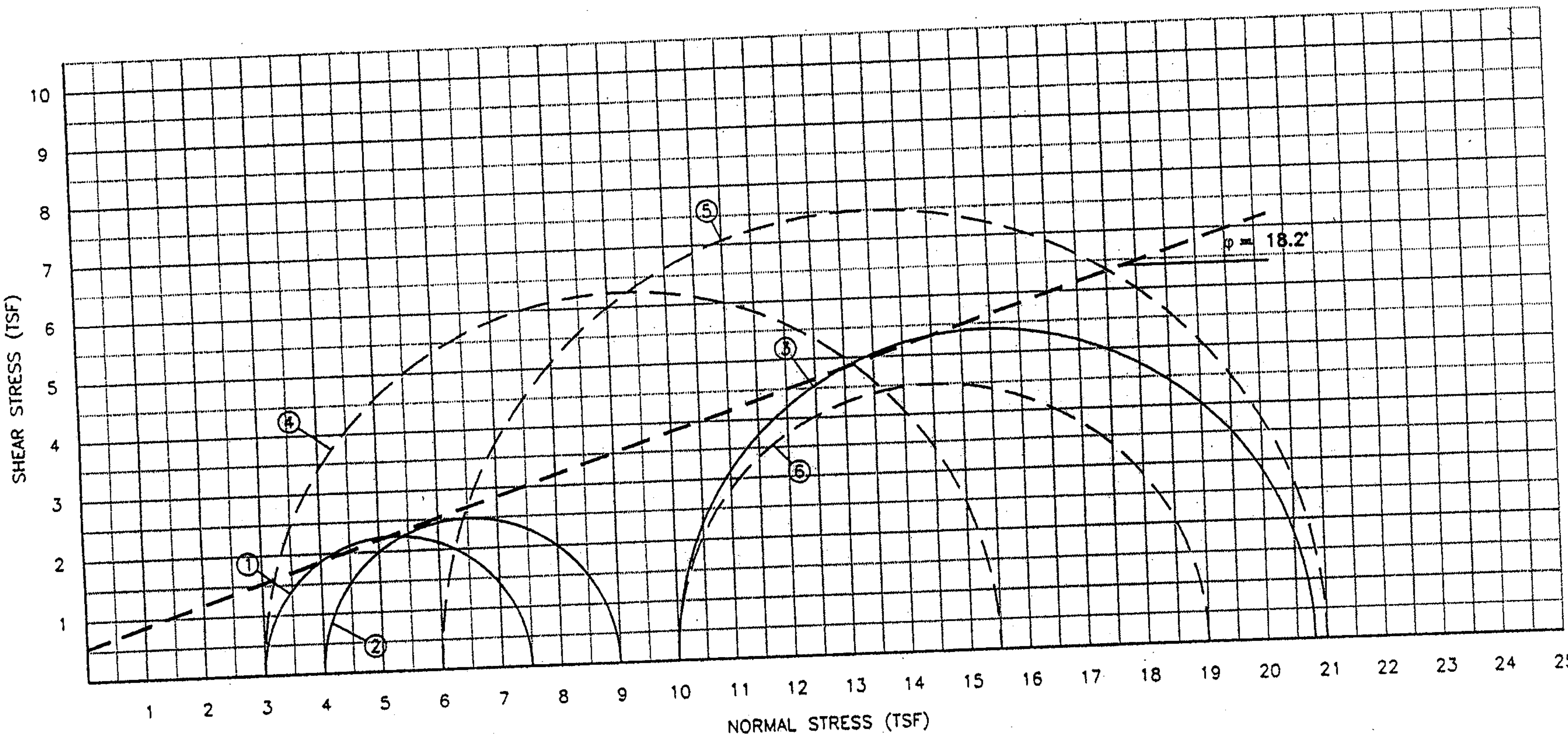
Illinois Licensed Professional Design Firm 184-001145

PDC Technical Services, Inc.
Peoria, Illinois

FIGURE 2
SHEAR TEST DATA
CLINTON LANDFILL NO. 3
CLINTON, ILLINOIS
PROJECT NO. 91-118

REVISED OCTOBER 25, 2006

CAD DWG NO:911181108 EMPLOYEE NO: 89736 DATE/TIME: 12/22/05 16:21 ACAD-15.



BERRY CLAY
CONSOLIDATED DRAINED
RESIDUAL STRENGTH
(Long-Term Behavior)

SAMPLE SOURCE

① EX-21 @39'
② EX-19 @59'
③ EX-13 @62'

RADNOR TILL
CONSOLIDATED DRAINED
RESIDUAL STRENGTH
(Long-Term Behavior)

SAMPLE SOURCE

④ EX-16 @72'
⑤ EX-19 @69'
⑥ EX-24 @76'

$c = 0.55$

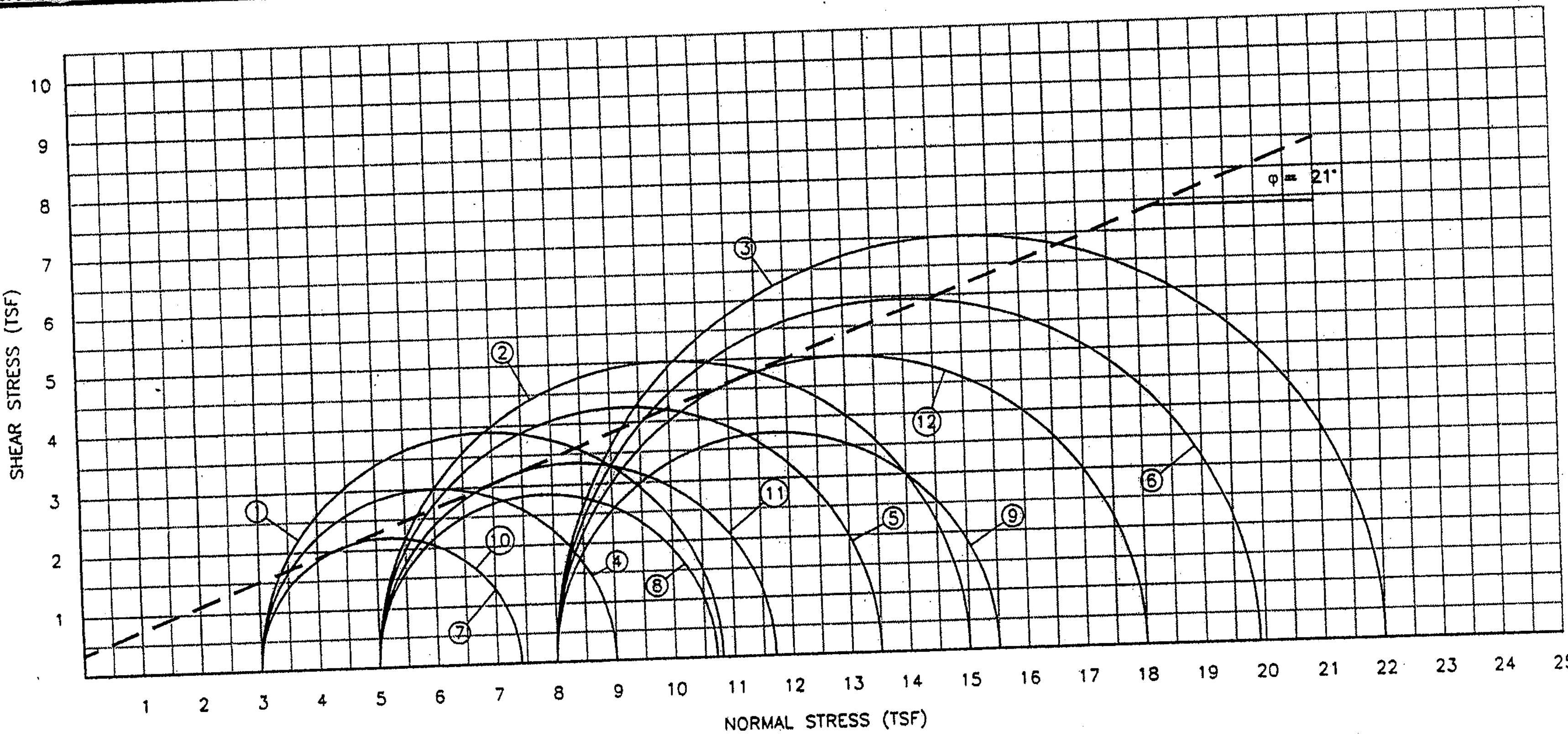
$\tan \varphi = \dfrac{4.5-.55}{12-0}$

$\varphi = 18.2°$

Illinois Licensed Professional
Design Firm 184-001145

PDC Technical Services, Inc. Peoria, Illinois	FIGURE 3
	SHEAR TEST DATA
	CLINTON LANDFILL NO. 3
	CLINTON, ILLINOIS
	PROJECT NO. 91-118

CAD DWG NO:9111811086 ... NO: 9834 DATE/TIME: 10/25/06 14:20 ACAD-16.



COMPACTION/GEOLOGICAL UNIT

1. 95%/TISKILWA (WEATHERED)
2. 95%/TISKILWA (WEATHERED)
3. 95%/TISKILWA (WEATHERED)
4. 95%/TISKILWA (UNWEATHERED)
5. 95%/TISKILWA (UNWEATHERED)
6. 95%/TISKILWA (UNWEATHERED)
7. 95%/BERRY CLAY
8. 95%/BERRY CLAY
9. 95%/BERRY CLAY
10. 95%/BERRY CLAY
11. 95%/BERRY CLAY
12. 95%/BERRY CLAY

COMPACTED EARTH LINER/AND COMPACTED CLAY FILL
CONSOLIDATED DRAINED
RESIDUAL STRENGTH
(Long-Term Behavior)

$c = 0.35$

$\tan \varphi = \frac{6.2-0.35}{15.0-0}$

$\varphi = 21^\circ$

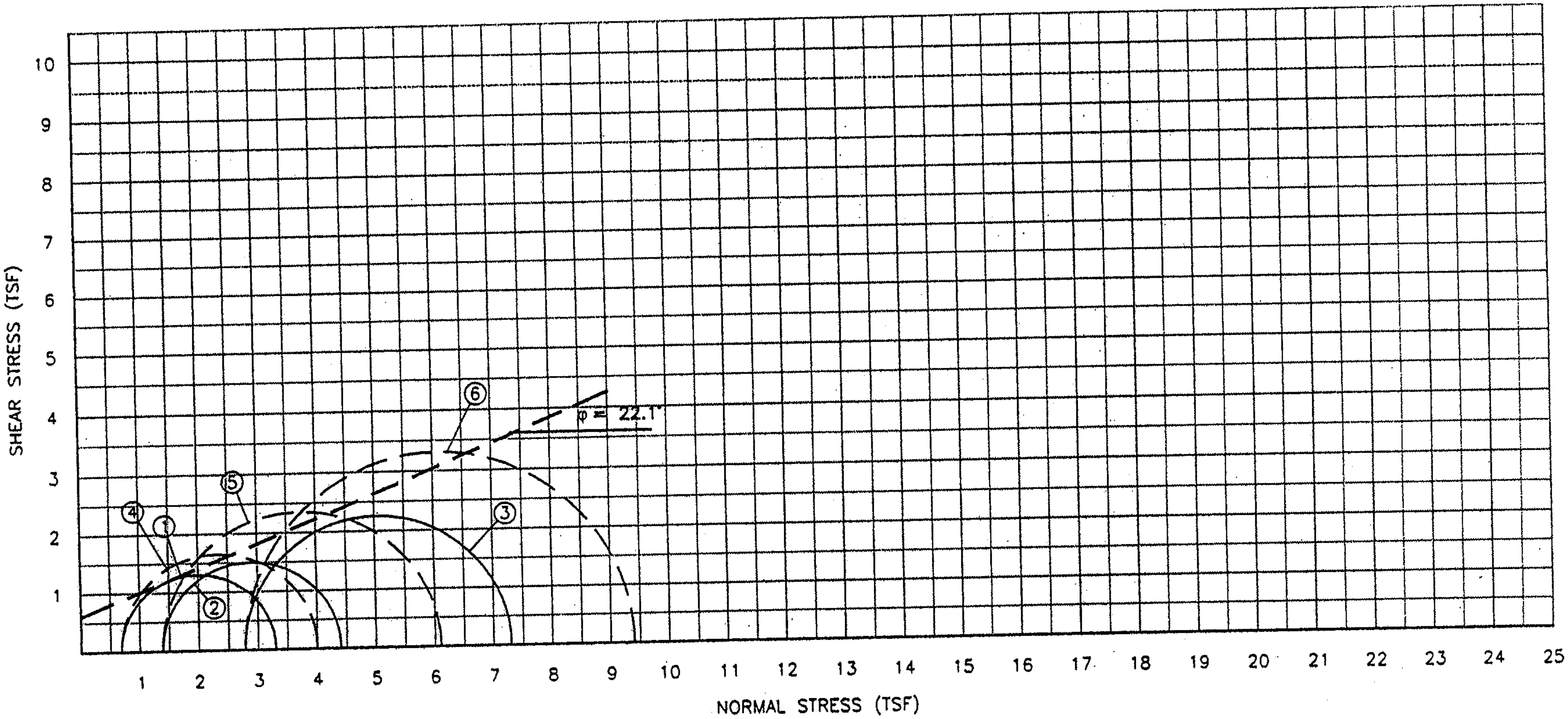
Illinois Licensed Professional Design Firm 184-001145

PDC Technical Services, Inc.
Peoria, Illinois

FIGURE 4
SHEAR TEST DATA
CLINTON LANDFILL NO. 3
CLINTON, ILLINOIS
PROJECT NO. 91-118

REVISED OCTOBER 25, 2006

CAD DWG NO:9111811086 ... O: 9834 DATE/TIME: 10/25/06 11:22 ACAD-16.



TISKILWA TILL (UNWEATHERED)	TISKILWA TILL (WEATHERED)
CONSOLIDATED DRAINED	CONSOLIDATED DRAINED
REMOLDED COMPOSITE	REMOLDED COMPOSITE
RESIDUAL STRENGTH	RESIDUAL STRENGTH
(Long–Term Behavior)	(Long–Term Behavior)

COMPACTION		COMPACTION	
①	87%	④	87%
②	87%	⑤	87%
③	87%	⑥	87%

$c = 0.65$

$\tan \varphi = \frac{3.1-0.65}{6-0}$

$\varphi = 22.1°$

Illinois Licensed Professional Design Firm 184-001145

PDC Technical Services, Inc.
Peoria, Illinois

FIGURE 5

SHEAR TEST DATA

CLINTON LANDFILL NO. 3

CLINTON, ILLINOIS

PROJECT NO. 91-118

REVISED OCTOBER 25, 2006

USGS Earthquake Hazards Program - Bedrock Acceleration

Shaw®

Project Name: Clinton TSCA Landfill
Date: Aug 14, 2007

Conterminous 48 States
2002 Data

Hazard Curve for PGA
Latitude = 40.1100
Longitude = -88.9600

Data are based on a 0.05 deg grid spacing
Frequency of Exceedance values less than
1E-4 should be used with caution.

Ground Motion (g)	Frequency of Exceedance (per year)
0.005	2.5222E-02
0.007	1.8332E-02
0.010	1.2952E-02
0.014	8.9304E-03
0.019	5.9856E-03
0.027	3.8931E-03
0.038	2.4264E-03
0.053	1.4096E-03
0.074	7.5878E-04
0.103	3.8141E-04
0.145	1.7596E-04
0.203	7.850E-05
0.284	3.4673E-05
0.397	1.5918E-05
0.556	7.8472E-06
0.778	3.9455E-06
1.090	1.8778E-06
1.520	8.0315E-07
2.130	2.5404E-07

Ground Motion (g)	Freq. of Exceed. (per year)	Return Pd. (years)	P.E. %	Exp. Time (years)
0.0981	4.2141E-04	2373.0	10.00	250.0